

REVIEW OF FHWA REPORT ON THE FAILURES ON I-795
NEAR GOLDSBORO NORTH CAROLINA
(PRELIMINARY)

By

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Root Pavement Technology, Inc.

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Executive Summary

The attached report reviews the findings of the Federal Highway Administration (FHWA) Report dated December 24, 2008 on the premature pavement failures experienced on a limited portion of Interstate 795 (I-795) near Goldsboro, North Carolina. The report is based on the information currently available (including the second forensic core testing program conducted by the North Carolina Department of Transportation (NCDOT) and S.T. Wooten that was not included in the FHWA study) and a site visit conducted in early February 2009. At the time of this report, some results from testing conducted by NCDOT were unavailable. This omitted data includes complete core test results for the first forensic study and the Falling Weight Deflectometer (FWD) data. The findings and conclusions contained in this report are subject to modification if and when additional information is provided.

A detailed review of the FHWA report indicates that the FHWA conclusions are not supported by the project data and that FHWA's determination of the cause of the limited areas of pavement failure on the project is incorrect.

Specific key findings of this review of the FHWA Report are as follows:

High air voids in the HMA surface layers were not a contributing factor to the limited early pavement failures found on the I-795 project as concluded by the FHWA report. Sections of I-795 identified by FHWA as having higher air voids are performing better than sections identified by FHWA as having lower air voids. Also, data from the Quality Control (QC)/Quality Assurance (QA) testing conducted during construction and data from the second round of forensic testing conducted by NCDOT contradict the air void data cited in the FHWA report.

Available data indicates that the bond strength between the surface layers of the asphalt is not deficient as suggested in the FHWA report and was not a contributing factor to the limited areas of early pavement failure. The bond strength failures as noted by NCDOT have occurred in areas of high pavement deflection.

Based upon my review of the data and my visual examination of the project, the following are the factors which likely contributed to the limited pavement failure on the I-795 project:

The sections of I-795 that were specified for 5.2 inches of HMA have insufficient pavement thickness to support the number and weights of vehicles using the roadway.

The lack of shoulder drains on the majority of the project and in areas of the failed sections where water is present in the ditches and land adjoining portions of the I-795 project created a bathtub effect and reduced the strength of the pavement structure.

The Superpave design specifications for the I-795 project resulted in a dry, brittle asphalt mixture which likely contributed to the severity and extent of premature cracking on certain portions of the project.

The NCDOT Roadway Density Assurance Program used to monitor and accept the contractor's testing during construction functioned as planned. This is demonstrated by the results of the second forensic core testing and the test results on the I-795 intermediate and base layers.

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This report reviews the findings of the Federal Highway Administration (FHWA) Report dated December 24, 2008 on the premature pavement failures experienced on Interstate 795 (I-795) near Goldsboro, North Carolina. The report is based on the information currently available (including the second forensic core testing program conducted by the North Carolina Department of Transportation (NCDOT) and S.T. Wooten that was not included in the FHWA study) and a site visit conducted in early February 2009. At the time of this report, some results from testing conducted by NCDOT were unavailable. This omitted data includes complete core test results for the first forensic study and the Falling Weight Deflectometer (FWD) data. The findings and conclusions contained in this report are subject to modification if and when additional information is provided.

BACKGROUND

I-795 was originally designated US-117. The approximately 18 mile long project consisted of a new alignment with a 4 lane divided highway with 12-ft lanes and 10-ft outside and 4-ft inside paved shoulders. The roadway was built utilizing separate contracts for earthwork and paving. Additionally, the paving work was carried out under two separate sets of plans (North Section – R1030F and South Section – R1030E). Some portions of the project were opened to traffic in December 2005.

The roadway as designed by NCDOT and built by S. T. Wooten included two different pavement structural sections. The far south end of the South Section (Mile Post (MP) 87.44 to 88.7) consists of 8.3 inches of Hot Mix Asphalt (HMA) on 8 inches of Aggregate Base Course (ABC) on subgrade. The remainder of the South Section and all of the North Section (MP 88.7 to 105.45) has a pavement section that consists of 5.2 inches of HMA on 11 inches of ABC on 7.9 inches of treated subgrade.

By the summer of 2007, limited areas of high-severity distress including cracking and rutting due to fatigue were observed in the 5.2-inch HMA section on the South Section. Significant repairs were completed on these areas in the fall of 2008 under a separate contract.

During a project inspection I conducted in February of 2009, some small areas in the North bound lane were experiencing cracking with materials from the base and/or subgrade coming to the surface (less than 500 feet long in the driving lane). Some minor surface cracking was also observed at various locations in the driving lane throughout the project (less than 5 percent of the total pavement surface). These areas were not continuous and no rutting could be detected.

NCDOT has spent considerable time and assets evaluating the pavement on the I-795 project. NCDOT has conducted a FWD pavement strength analysis, at least two forensic studies where cores were taken and tested, and several other investigations to determine the cause(s) of the

failures.

PAVEMENT FAILURE

The visual elements of the limited pavement failure on I-795 reflected in the picture below (alligator/block cracking in the wheel path along with rutting) are classic manifestations of pavement failure due to fatigue (Photograph 1).

Photograph 1
Southbound Lane Prior to Repair



The white material outlining the cracking is fine material coming up from the ABC and/or subgrade under the HMA layers. According to the FHWA report, NCDOT observed water coming up through the pavement to the surface. It appears that the pavement is rutted in the cracked area. The section of roadway depicted above has been repaired under a separate contract with NCDOT during the Fall of 2008. I saw no similarly failed areas on the project during my site visit of February 2009.

Fatigue failure is the result of a pavement structure receiving more loading (Equivalent Single Axle Loads (ESALs)) than it is capable of carrying. The pavement with the 8.3-inch HMA layer on the south end of the South Section shows no signs of distress. The same is true for the adjoining projects paved with a similar structural section of approximately 8 inches of HMA.

PAVEMENT STRUCTURAL DESIGN

The FHWA report concludes: "The pavement design was according to NCDOT procedures. The inputs used in the pavement evaluation were reasonable and the structural design was predicted to be satisfactory for the assumed design traffic."

The design inputs used by NCDOT resulted in a required Structural Number (SN) of 4.3. The pavement SN calculated by FHWA for the short section with the thicker 8.3-inch HMA layer was 4.68, and the pavement SN for the majority of the job with the 5.2-inch HMA layer was 4.78. Both values exceed the NCDOT 20 year design requirement. A comparison of the two Structural Numbers indicates that the 5.2-inch HMA pavement should be slightly stronger than the 8.3-inch HMA pavement. However, FWD data conducted by NCDOT indicates that the 8.3-inch HMA section is significantly stronger than the rest of the pavement, particularly in the southbound lane where the majority of the failures have occurred.

The structural design procedure utilized by NCDOT assumed that each layer obtains its design strength during construction, maintains that strength for the life of the pavement and that the entire thickness develops the design strength. A study in Alabama have indicated that ABC layers above a thickness of approximately 8 inches do not contribute any additional strength to the pavement structure. Based on the Alabama study, the pavement section designed with 11 inches of ABC would perform like a pavement with 8 inches of ABC. This would reduce the structural number for the 5.2-inch HMA section from 4.78 to 4.36, which in part indicates that the failures should occur in 5.2-inch HMA pavement section first, which was what NCDOT and FHWA observed.

PAVEMENT DRAINAGE

One design requirement necessary to insure that the ABC layer and subgrade maintain strength during the life of the pavement is to prevent water from entering the pavement structure and to drain water out if it gets in. The original NCDOT design called for shoulder drains in limited locations of the project.

The North Section called for 14.211 km of 4-lane pavement, which totals 56.8 km of pavement shoulders. The NCDOT plans called for a total of 11.75 km of shoulder drains for this part of the project, covering only 20.7 percent of the total shoulder length.

The South Section called for 14.793 km of 4-lane pavement, which totals 59.2 km of pavement shoulders. The plans called for a total of 10.9 km of shoulder drains for the South Section, covering only 18.4 percent of the total shoulder length.

Table 1 shows a summary of the four locations repaired in 2008 on the South Section and the shoulder drains specified in the contract for each of the repaired areas.

Table 1
Shoulder Drains in Repaired Areas

Lane	Repair Start – MP (Station)	Repair End – MP (Station)	Drains – Outside Station	Drains – Median Station
Northbound - Right	88.2 (79+72)	89.1 (94+21)	87+50 to 92+20	None
	94.3 (17+793)	95.2 (19+242)	17+730 to 18+320 19+040 to 19+212	19+225 to 19+270
Southbound - Left	89.5 (102+10)	92.3 (161+30)	119+40 to 127+20 143+40 to 149+80 158+60 to 162+20	None
	94.8 (18+598)	95.0 (18+920)	None	18+326 to 18+620

Table 2 summarizes the total length of each of the repairs and the length and percentage of shoulder drains at the locations where the pavement was repaired.

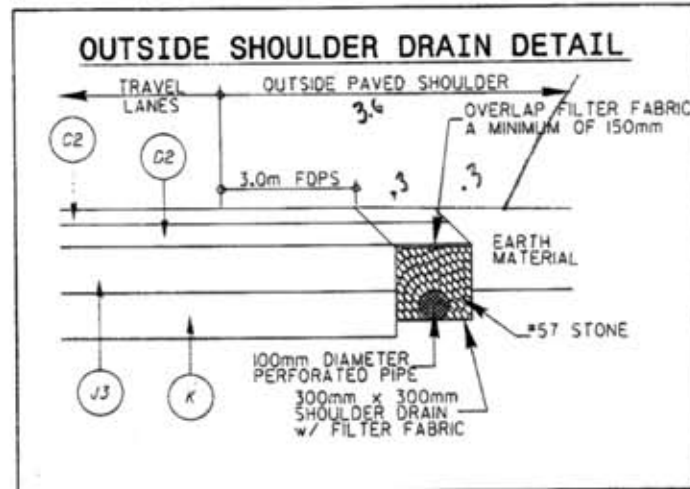
Table 2
Length (Percent) of Shoulder Drains in the Repaired Areas

Lane	Repair Start – MM (Station)	Repair End – MM (Station)	Length of Repair - m	Drains Outside - m (%)	Drains Median - m (%)
NB	88.2 (79+72)	89.1 (94+21)	1,449	470 (32.4)	None (0)
	94.3 (17+793)	95.2 (19+242)	1,449	699 (48.2)	45 (3.1)
SB	89.5 (102+10)	92.3 (161+30)	5,920	1,690 (28.5)	None (0)
	94.8 (18+598)	95.0 (18+920)	322	None (0)	22 (6.8)

The combined outside and median shoulder drains for the four failed areas is less than the average percentage of drains (19.5%) specified for the North and South Sections.

Pavement sections built without shoulder drains have a much greater chance for poor performance. This is demonstrated in NCDOT's plan detail drawing for shoulder drains for the South Section of the project (Figure 1). In the drawing, "K" represents the treated subgrade material and "J3" the 11 inches of ABC. The "earth material" shown to the right of the drain is a soil material intended to grow grass and resist erosion due to rainfall.

Figure 1



When the shoulder drain is not present, the "earth material" acts as a dam and holds the water in the pavement structure, creating a bathtub effect. Visual observation made during my February 2009 site visit indicates water standing and/or running in the ditches and woods near the areas of pavement failure. The possibility of trapped water in the pavement could explain the low strength values measured in the failed areas by the FWD testing.

FWD STRENGTH ANALYSIS

The FWD analysis conducted individually by NCDOT and FHWA showed a weakened structure under the HMA layers. NCDOT concluded the following: "All subgrade modulus values calculated from the FWD data are above 10,000 psi, considered the cutoff for adequate subgrade modulus values". The NCDOT result measured with the FWD does not agree with the original NCDOT pavement design California Bearing Ratio (CBR) of 10.8, which equates roughly to a modulus of 16,200 psi. The lower effective subgrade strength would require a design that increases HMA pavement thickness by approximately 21 percent. If the pavement thickness is not increased, the effect on the pavement design life would be to reduce the pavement life from the planned 20 years to approximately 2.5 years.

The FHWA report indicates that: "An average MR of about 24,000 psi seems to be representative of the foundation support of the HMA layers". Based on standard engineering practice, a responsible designer would not design a pavement structure using an average modulus value. Typically, values at the 85 percentile (meaning that only 15 percent of the test results are lower) would be used for design. This concept would yield a modulus value of approximately 20,000 psi, which equates to a CBR of approximately 13.3. The FHWA's FWD strength number of 20,000 psi (or CBR 13.3) reflects a composite strength of pavement layers that should have been significantly stronger than the actual composite strength of the pavement on the I-795 project: (1) an 11 inch layer of ABC which should have had a CBR between 100 and 150, (2) an 8 inch treated subgrade layer which should have had a CBR of over 25 and (3) the subgrade under the pavement structure. Using the FHWA data, the pavement structure can be considered a full depth pavement (HMA layer on top of uniform subgrade with a CBR of 13.3). The pavement structure required for this section should be 44 percent thicker than the current 5.2-

inches of HMA in place. Based on the FHWA data, without increasing the present thickness of the HMA, the pavement life of I-795 would be approximately 6-months.

The above analyses are estimates presented for general information and trends only. They are based upon the lower pavement support values found by NCDOT and FHWA in the weaker areas and do not necessarily represent the pavement structural values for the entire project.

PAVEMENT TRAFFIC ANALYSIS

NCDOT conducted an onsite traffic analysis for the project in 2007 and found the total numbers of vehicles were fewer than predicted, however NCDOT found a significantly higher percentage of the total vehicles were trucks and that 10% of the 5-axle trucks were overloaded.

NCDOT stated that: "The traffic count data yielded 5.93 million Equivalent Single Axle Loads (ESALs) over a 20-year design period while the traffic forecast data yielded 8.54 million ESALs over a 20-year design period". It is unclear to what extent NCDOT took the overloaded vehicles into account in making its ESAL predictions. Overloaded vehicles cause a great deal more pavement damage than vehicles traveling at the legal weight limit. The FHWA report acknowledges that the amount of overweight truck traffic could have contributed to the early limited pavement failures observed on the I-795 project, a point with which I concur.

The FHWA report commented on the same data: "The data shows that design traffic loading and actual traffic loading is similar." This disagrees with the NCDOT findings that the actual traffic loadings on I-795 were over 30 percent less than the design predictions.

AIR VOIDS IN PAVEMENT SURFACE LAYERS

The FHWA report concludes that the pavement thickness design developed by NCDOT was adequate and that the current pavement strength as measured by the FWD forensic analysis is insufficient to carry current traffic. FHWA further concludes that the poor pavement performance is caused by high air voids in the two surface layers as measured by the first forensic investigation conducted by NCDOT. Specifically, the FHWA report concludes: "Throughout the project, the two HMA surface course layers have air voids greater than expected in design. The high air voids are a contributing factor leading to the premature pavement distress". The FHWA assumes that high air voids in the approximately 3 inch thick HMA surface layers have reduced the strength of the surface layers from that anticipated in the pavement thickness design calculations and therefore have caused the pavement to fail prematurely.

The following results indicate that the FHWA determination of high air voids in the surface layers is at least biased high and most likely flawed.

First Forensic Study - Several factors have led to the mistaken determination of high air void test results indicated by the first set of NCDOT forensic tests conducted on cores from the project. Specific problems with the data are as follows:

1. I understand that Rice Maximum Specific Gravity (Gmm) tests were conducted on broken up core samples from the I-795 roadway, and that no effort was made to remove the aggregate particles cut by the coring process. The test procedure requires that a “dry back” procedure be used to correct for any water absorbed into the aggregate during the test. Under normal mix design and construction testing, the amount of water absorbed would have little effect on the results. However, the chance of absorbing water into the aggregate is greater when testing mixture samples from forensic cores, and the calculated air void results should be reduced between 0.5 and 1 percent or more to accommodate for the water picked up during testing. The forensic data does not indicate that this adjustment was made.
2. The FHWA report states that the forensic cores were tested using the CoreLok method. The CoreLok procedure places the samples in a plastic bag and attempts to remove all the air from the bags with a vacuum pump. The bags are sealed and then the sample volume is measured. A National Center for Asphalt Technology (NCAT) study has found that the CoreLok procedure incorrectly measures the volume of the sample, which results in higher air voids because of an inability to get the plastic bag into all the rough textured voids on the core surfaces (Transportation Research Board Record 1891 – Xie and Watson, 2004). The NCAT study recommends using a correction factor to reduce the measured air voids by 0.5 percent. The forensic data does not indicate that this adjustment was made.
3. I understand that the Florida Shear Test was used to determine bond strength at the surface layer interface and this by necessity was conducted prior to density testing of the pavement cores. Recently, NCAT reviewed various bond strength testing procedures, including the Florida method, to develop a standardized testing method (NCAT Report 05-08). The proposed NCAT method requires a minimum of a 2-inch layer (both top layer and bottom layer) for samples to be tested with their procedure. The load required to separate a 6-inch diameter core is in excess of 2,200 pounds for an 80 psi bond strength. With the 1.5-inch surface layers used on the I-795 project and the high loads necessary to separate most of the cores, it is highly possible that the surface layer cores were damaged during the Bond Strength testing.
4. For the first forensic study NCDOT cored the pavement at selected locations; it is not known how NCDOT determined which locations to core. NCDOT obtained four cores at approximately 27 sites based on counting bond strength results on Figure 20 of the FHWA report. Cores A and C were taken in the outside wheel track of the driving lane; cores B and D were taken between the wheel tracks in the same lane. Photograph 2 shows the location and close proximity of the 4 samples taken for core sample 40. All the core sites for the first forensic study were sampled the same way.

The FHWA report, for reasons unknown to me, did not include any test results for core location 40. In fact the FHWA report only provided test results for 13 out of the 27 (approximate number determined as previously stated) core sample sites from the southbound lane and a similar ratio for cores taken from the northbound lane. The FHWA report does not explain why FHWA did not report air void data from over half

Photograph 2
Core Sample 40 - First Forensic Study



the cores taken in the first forensic study.

Photograph 2 demonstrates that the A and C cores and B and D cores were taken close together on the roadway. Given the proximity of the cores, one would expect the air void results to be similar. However, some of the individual data does not reflect these expectations. For example, at core site 120 in the southbound lane, the A and C cores from the outside wheel track had air voids of 9.0 and 5.6 percent, respectively. Since these cores were taken within inches of each other, this is physically impossible based on the type of construction equipment and methods of placement called for in the specifications and used on the project by the contractor. The B and D cores taken from between the wheel tracks show the same confusing results with air voids of 10.4 and 6.3 percent, respectively.

A second anomaly in the data is the major difference between the air voids of the cores taken from the wheel track and those from between the wheel tracks. Normally, wheel track cores have lower air voids when compared to cores taken from between the wheel track because traffic compacts the layers in the wheel track. In the southbound driving

lane, there were seven cores reported from mixture produced at the S.T. Wooten Sims plant and six cores reported from mixture produced at the S.T. Wooten Princeton plant. Four of the seven Sims plant HMA test locations had air voids between the wheel tracks significantly lower than in the wheel track. It is extremely unlikely that the construction equipment and methods of placement called for in the specifications and used on the project by the contractor could possibly build this type of air void difference into two sets of samples taken within a few feet of each other. For example, the test results from Location 168 had an average of 8.8 percent air voids in the wheel track (cores A and C) and an average of 7.3 percent air voids between the wheel tracks (cores B and D).

For the six core locations taken from the Princeton plant HMA mixture in the southbound lane, the same deviation existed at one core location and the exact opposite deviation (wheel track air voids lower than between wheel track) occurred at three of the core locations. For example, the test results from Location 152 had an average of 9.4 percent air voids in the wheel track (cores A and C) and an average of 13.4 percent air voids between the wheel tracks (cores B and D). Again, construction equipment cannot generate this discrepancy, and traffic cannot compact Superpave mixtures to a degree that would support such results.

Based on the testing bias, possible core damage from the bond strength test and the highly unlikely test results at a number of locations, the air voids results for the first set of forensic testing should be considered highly suspect.

Second Forensic Tests – In the second forensic study, NCDOT identified 18 locations in the project where NCDOT and S. T. Wooten had built and tested nuclear control strips to establish compaction procedures and to calibrate the nuclear gauges to roadway cores. The gauge calibration process required taking five randomly located cores from the control strip after both NCDOT and S. T. Wooten have taken two readings with their individual nuclear gauges at each core location.

For the second forensic study, six cores were taken from each control strip location as shown in Photograph 3. Two cores (1 and 4) were tested by S. T. Wooten using two test methods, two cores (2 and 5) were tested by the NCDOT project staff using four test methods and two cores (3 and 6) were tested by the Materials and Test staff in Raleigh using four test methods. A summary of the data developed from the three labs along with a comparison to the original QC and/or QA results measured during construction by both NCDOT and S.T. Wooten, are presented in Appendix A for the “First Surface Layer” and “Final Surface”.

One of the 18 locations tested (location 16) in the southbound outside shoulder did not indicate which layer the control strip was conducted on. Therefore this data was not included in the results in Appendix A. Twelve locations were sampled from the median lane, three locations were sampled from the outside lane and two locations were sampled from the outside shoulder.

For the 17 sections included in the analysis, 12 locations from the second forensic testing had air voids that essentially met the NCDOT specification requirement of 92 percent compaction in the

Photograph 3
Core Sampling SB Median Lane for Second Forensic Study



completed pavement (8 percent air voids). Only one location (13 at Mile Post 97.75) had forensic results (11.8 percent air voids) that were outside of reasonable tolerances when compared with the QC and QA construction results (6.6 percent air voids). Four other locations had forensic air voids that agreed with QC/QA data collected by NCDOT and the contractor during the I-795 project.

Comparison of First Forensic Air Voids to Pavement Performance - Figures 20 and 21 in the FHWA report (reproduced below), show that air voids have no relation to pavement performance. For the southbound lane (Figure 20), the “final surface” layer air voids are higher from MP 95 to MP 105, an area of no repairs when compared to the air voids at the south end of the project, where repairs were made. For the “first surface” layer from MP 95 to MP 105, some air voids are higher and some lower than the air voids in the repair areas at the south end of the project.

Results in the northbound lane are similar. The air voids in the “final surface” layer are significantly higher in the non-repair areas from MP 93 to MP105 compared to the south end of the project where the repairs were made. The air voids in the “first surface” layer are slightly

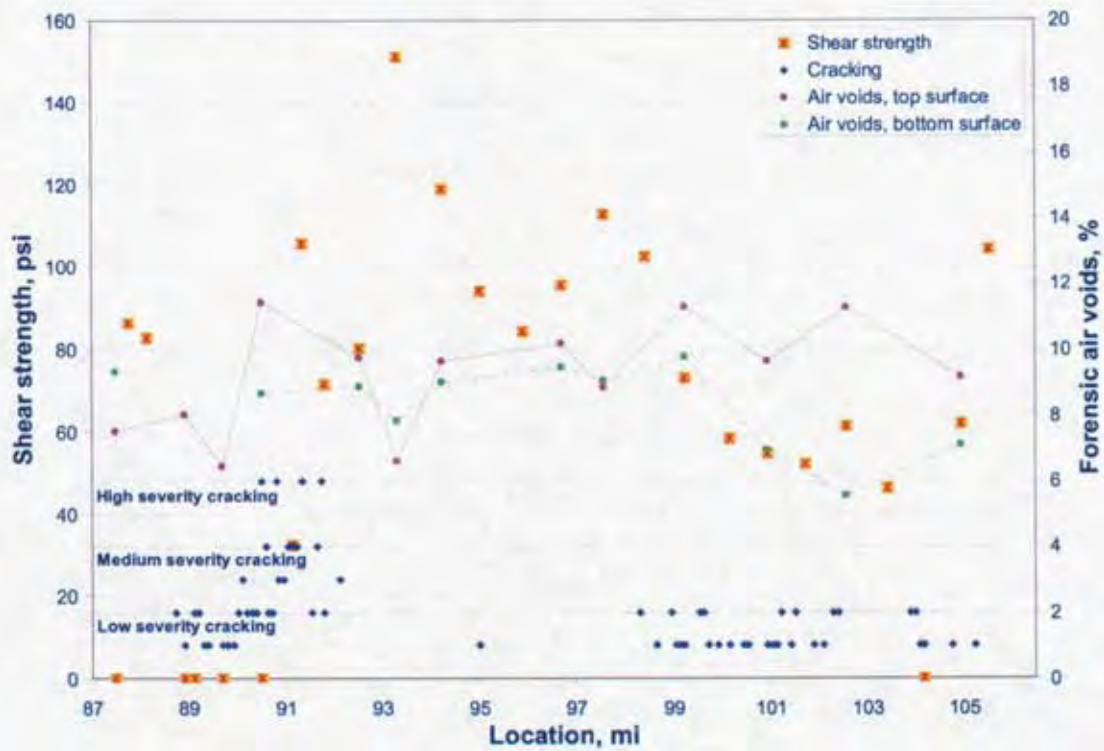


Figure 20. SB I-795 pavement condition and HMA surface layer air-voids.

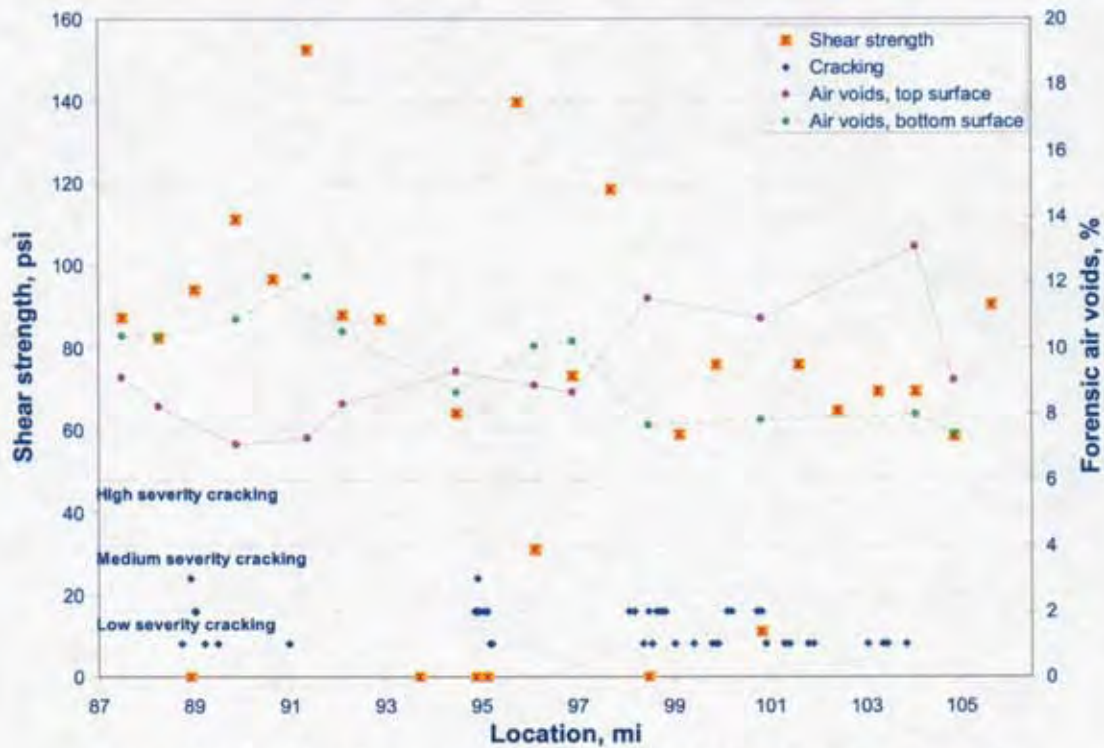


Figure 21. NB I-795 pavement condition and HMA surface layer air-voids.

lower in the non-repair areas.

The FHWA premise that the pavement fatigue failures were primarily caused by high air voids in the surface layers is flawed. Based on the FHWA data, one would expect to have the same or more severe pavement failures at the north end of the project where the air voids are higher. This inconsistency, combined with the bias in the first forensic cores testing procedures and the results of the second forensic core study, leads to the conclusion that the air voids in the pavement surface layers did not contribute in any significant way to the limited pavement failures on the I-795 project.

The current condition of the pavement surface on I-795 further indicates that the test results for the first forensic testing were flawed. Pavement with in-place air voids of 12 percent or more as reported in the FHWA study would exhibit significant areas of pavement raveling after three plus years of traffic and/or exposure to the environment. However, during the review of pavement performance in February 2009, no significant pavement raveling could be detected.

BOND STRENGTH TESTING PROGRAM

As discussed in a prior section, the surface layers were too thin for proper bond strength testing as conducted by NCDOT. NCDOT's use of thin samples in the bond strength testing equipment likely increased the testing variability.

The FHWA analysis seems to distort the actual data in the following ways:

1. In its analysis, the FHWA included all the core samples that were debonded in the roadway or debonded during the coring process as having failed the bond strength test. NCDOT stated that: "only cores taken from the distressed area show delamination of the top surface layer". If debonding of the pavement surface was caused by a flaw in the construction process, then it would appear throughout the project. However, this was not the case. As noted by NCDOT, the debonding of the surface layer only occurred in areas of high deflection, which most likely caused the bonds to break.
2. The bond strength data shown in Figures 7 and 8 in the FHWA report indicate that only two or three test results out of more than 20 samples in each of the lanes were significantly below the minimum desired strength of 60 psi. Figures 7 and 8 also show that the bond strength of the pavement cores from the north end of the project (that part of the project with no repairs) is significantly lower for both lanes than the south end of the project where the limited failures occurred and were repaired.
3. The FHWA correctly identifies Ground Penetrating Radar (GPR) as an experimental technology that is not yet proven. In light of this conclusion, with which I agree, and the information in paragraphs 1 and 2 above, I believe it was imprudent for the FHWA report to rely on GPR testing to conclude that the pavement had a relatively high risk of debonding throughout the project.

Based on the bond strength data presented, the pavement was built with reasonable bond strength, and the pavement bond between the layers had no effect on the limited failures on the I-795 project.

NCDOT QUALITY ASSURANCE PROGRAM

The FHWA report critiques the NCDOT Quality Assurance Program by stating: "It is recommended that the NC DOT in-place HMA field density program be reviewed. For in-place HMA field density determination, an adequate HMA Gmm and Gmb verification testing program needs to be provided. The program needs to include NC DOT ensuring the samples are randomly taken and NC DOT taking possession and storage of the samples through the testing process. Alternative Gmb test methods should be explored that adequately characterize the HMA quality". This criticism is based on the FHWA belief that the current NCDOT QA program failed to identify the allegedly high air voids in the two surface layers. FHWA has failed recognize the many flaws in the first forensic testing program, the results of the second forensic testing and the fact that the NCDOT Roadway Density QA functioned as planned on the base and intermediate layers on the I-795 project.

SUPERPAVE ASPHALT MIXTURES

In the past, the Marshall method was used to design mixtures in the laboratory. The mixtures were designed to include 4 percent air voids, with the knowledge that they would be placed in the field at approximately 8 percent air voids and then be compacted by traffic to the desired 4 percent air voids to provide the long term mixture durability.

With the laboratory compaction procedures developed for the Superpave mixtures, such as the mixture designs required by NCDOT for the I-795 project, this is no longer the case. A recent study by NCAT has shown that the high gyration Superpave mixtures do not have any significant increase in compaction (reduction of air voids) as a result of traffic (NCHRP Report – 573).

The primary motivator in the development of the Superpave process was to design mixtures that would resist pavement rutting in the asphalt layer. To do so, the researchers selected a high number of compaction gyrations for the mixture design process, resulting in dry and brittle pavement mixtures.

Over the past several years, several states have attempted to improve the flexibility and durability of their Superpave mixtures by lowering the number of design gyrations and increasing the minimum design voids mineral aggregate (VMA) in the mixture. I understand that NCDOT has recently begun taking steps in this direction.

The Superpave mixtures designed according to the NCDOT specifications and procedures for the I-795 project were relatively dry (low in asphalt content) and lifeless products with relatively poor flexibility (the ability to deflect without cracking). While the NCDOT's mix design specifications may not have been a primary cause of the pavement failures, the resulting designs have contributed to the severity and extent of the limited failures.

CONCLUSIONS

High air voids in the HMA surface layers did not contribute to the limited early pavement failures found on the I-795 project. Sections of I-795 identified by FHWA as having higher air voids are performing better than sections identified by FHWA as having lower air voids. Also, data from the Quality Assurance/Quality Control testing conducted during construction and data from the second round of forensic testing conducted by NCDOT contradict the air void data cited in the FHWA report.

Available data indicates that the bond strength between the surface layers of the asphalt is not deficient as suggested in the FHWA report and was not a contributing factor to the limited areas of early pavement failure. The bond strength failures as noted by NCDOT have occurred in areas of high pavement deflection.

Based upon my review of the data and my visual examination of the project, the following are the factors which likely contributed to the limited pavement failure on the I-795 project:

The sections of I-795 that were specified for 5.2 inches of HMA have insufficient pavement thickness to support the number and weights of vehicles using the roadway.

The lack of shoulder drains on the majority of the project and in areas of the failed sections where water is present in the ditches and land adjoining portions of the I-795 project created a bathtub effect and reduced the strength of the pavement structure.

The Superpave design specifications for the I-795 project resulted in a dry, brittle asphalt mixture, which likely contributed to the severity and extent of premature cracking on certain portions of the project.

The NCDOT Roadway Density Assurance Program used to monitor and accept the contractor's testing during construction functioned as planned. The results of the second forensic core testing and the test results on the I-795 intermediate and base layers support the conclusion that it was effective.

Appendix A

First Surface Layer

North Bound Lane

Core	MP	Lane	Construction			Forensic QC	QA			M&T
			QC	QA	Ave		1-VD 4-10M	2-VD 2-UCD 5-10M	5-T166	
1	87.75	M	8.9	8.3	8.60	9.1	9.3	9.6	10.9	9.9
									10.4	10.9
									10.9	10.20
2	90.27	M	7.4	-	7.40	7.2	6.7	6.5	7.4	6.8
									8.0	8.0
									8.0	7.14
4	93.28	M	6.2	-	6.20	6.7	7.1	6.7	7.6	6.7
									9.6	9.7
									9.6	7.74
7	96.45	M	8.9	8.2	8.55	7.1	6.6	7.0	7.5	7.5
									8.0	7.6
									7.6	7.28
8	97.75	M	7.3	-	7.30	8.6	7.8	8.1	9.0	7.4
									7.8	8.0
									8.0	7.9
									8.2	8.02
9	103.35	OS	7.9	6.2	7.05	5.9	5.9	5.8	6.5	6.0
									5.7	6.3
									6.5	6.00

South Bound Lane

Core	MP	Lane	Construction			Forensic QC	QA			M&T
			QC	QA	Ave		1-VD 4-10M	2-VD 2-UCD 5-10M	5-T166	
11	102.20	OS	8.6	-	8.60	7.8	7.6	8.0	8.9	7.5
									8.6	8.3
									8.2	8.2
									8.2	8.10
12	99.55	O	7.9	7.0	7.45	10.0	7.0	7.2	8.3	6.8
									7.2	7.3
									8.3	8.4
									8.4	7.77
18	87.70	M	7.5	7.7	7.60	7.0	7.8	8.6	9.7	8.0
									8.4	8.5
									8.4	9.3
									8.4	8.43

Final Surface Layer

North Bound Lane

Core	Construction					Forensic		QA		M&T					
	MP	Lane	QC	QA	Ave	1-VD 4-10M	2-VD 2-UCD5-10M 5-T166	3-10M 3-T166 6-ACD6-UCD	Ave						
3	93.18	M	7.3	7.4	7.35	7.9	8.3	8.2	8.6	8.1	8.3	8.0	8.5	8.8	8.35
5	93.60	M				10.9	10.6	10.7	11.7	11.1	11.8	10.4	11.2	12.1	11.29
6	93.75	O	8.1		8.10	7.2	7.1	7.0	7.8	6.7	6.9	7.1	7.4	7.4	7.21

South Bound Lane

Core	Construction					Forensic		QA		M&T						
	MP	Lane	QC	QA	Ave	1-VD	4-10M	2-VD	2-UCD	5-10M	5-T166	3-10M	3-T166	6-ACD	6-UCD	Ave
10	105.30	M	7.3		7.30	6.3	6.3	6.5	6.8	6.7	6.8	6.6	6.7	7.0	7.0	6.67
13	97.75	M	6.9	6.4	6.65	11.2	11.2	11.0	12.0	11.4	11.5	11.8	12.8	12.4	12.8	11.81
14	93.60	M	10.0	10.6	10.30	10.0	10.7	11.0	12.0	11.3	11.6	11.8	12.8	12.2	12.1	11.55
15	93.60	M	9.4		9.40	10.7	9.9	10.5	11.2	10.4	10.5	10.7	11.5	10.9	11.0	10.73
17	89.75	O	5.3	5.4	5.35	6.0	5.9	5.8	6.1	6.0	6.0	6.6	6.1	6.0	6.4	6.09